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Novel Cold-formed Steel Elements for Seismic Applications

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Novel cold-formed steel elements for seismic applications

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Abstract

Novel cold-formed steel (CFS) elements are investigated in this paper for seismic resistant multi-storey moment frames. Premature local buckling and low out-of-plane stiffness are known as the main structural deficiencies of CFS sections with thin-walled elements. These lead to low energy dissipation capacity of the structures made up of CFS sections as the main load bearing members in seismic events. In order to improve the energy dissipation capacity of CFS members, an innovative CFS beam section with curved flanges is developed by numerical FE analysis and experimental work. A web bolted through plate CFS beam-column connection is used to limit out-of-plane actions in transferring the beam forces to column faces. This type of connection, however, produces premature web buckling and needs to be strengthened by a combination of vertical and horizontal out-of-plane stiffeners. Six beam-column connection assemblies including different stiffener configurations were tested. It is shown that the ductility factor and the moment strength are increased by up to ~75% and ~35% respectively relative to the specimen without stiffener. Correspondingly, activation of connection slip leads to a highly stable hysteretic behaviour and a significant increase (up to ~240%) in the hysteretic energy dissipation capacity.

Introduction

The use of cold-formed steel (CFS) structures as main load bearing members is mainly limited to stud-wall frames with low ductility capacity (Moghimi et al, 2009; Casafont et al, 2007). Premature local buckling and low out-of-plane

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stiffness of the thin-walled elements of CFS structures are the main well known deficiencies. In a recently developed CFS moment resisting frame for single-storey dwellings (Sato et al, 2009; AISI S110, 2007), the ductility is mainly met by slip-bearing action within the bolted beam-column connections, while the beams and columns remain intact. In order to extend the use of CFS elements as the main members of multi-storey skeleton moment frames in seismic areas, there is a need to develop energy dissipation in CFS beams rather than just yielding the material around the bolt holes.

This paper presents analytical investigations and experimental work on novel CFS beam sections and appropriate beam-column connection details for seismic applications. The aims are to enable CFS beams to produce plastic moment sustained at large rotations similar to Class 1 cross sections in Eurocode 3 (EN 1993-1-1: 2005) and larger than 0.04rad required for Special Moment Frames (SMFs) in AISC Seismic Provisions (ANSI/AISC 341-05, 2005).

FE results for development of a high performance CFS beam

The large width/thickness ratios of the thin-walled elements of CFS sections avoid development of full plastic moment in the beams due to local buckling. The width/thickness limitations specified in the codes of practice (ANSI/AISC 360-05; EN 1993-1-1: 2005) cannot be easily met by typical CFS sections such as flat flange channels. Therefore, an evolutionary process was used to overcome the width/thickness limitation by introducing flange bends in channel sections leads to a curved flange section (Bagheri Sabbagh et al, 2012). FE analysis employing Abaqus (2007) was used to investigate the moment-rotation behaviour of 2m cantilever beams (representing a 4m span in a laterally loaded moment frame) with double back-to-back channel sections and different flange bends (Fig. 1, F0-4 and C sections).

The normalised fixed end moment- tip rotation curves (M/M_p-θ in which M_p= 58kNm, the nominal plastic moment of beam F0 with F_y= 275MPa) obtained in the FEA (Fig. 1) show gradual improvements for the beams with the same amount of material. The double back-to-back channel section without any flange bends (Fig. 1, F0) produced the moment resistance limited to $0.67M_p$ due to the premature flange buckling. By adding flange lips (Fig. 1, F1) both moment resistance and ductility were improved significantly. Further improvement was achieved in the moment-rotation behaviour by using intermediate flange stiffeners (Fig. 1, F2), which also prevented the sudden loss of strength occurred before 0.02rad for beam F1.

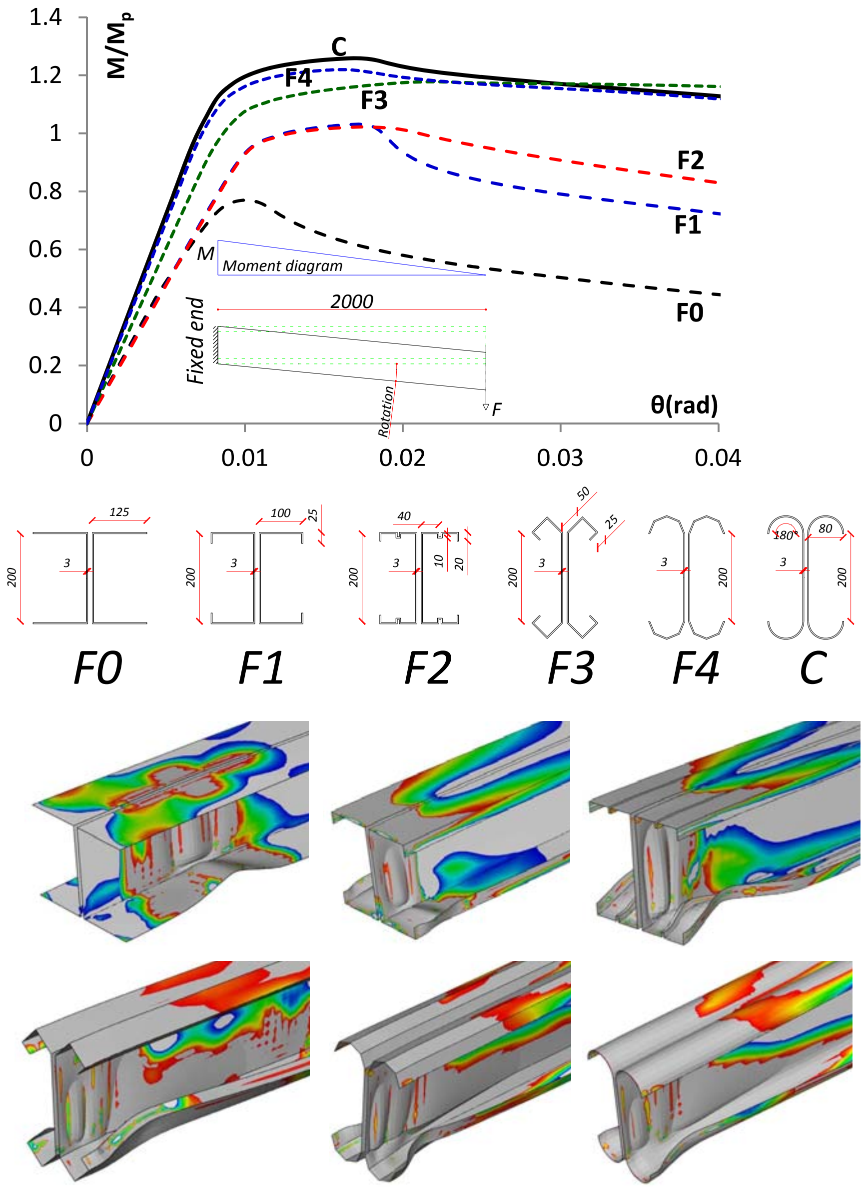

Fig. 1 Moment-rotation curves and failure deformation of 2m cantilever beams with F0-4 and C sections

An alternative to limit the width/thickness ratio introduces intermediate bends in the flanges (Fig. 1, F3). This solution not only increases the moment resistance and ductility capacity, but also increases the initial stiffness which can be of importance for moment frames. Increasing the number of flange bends (Fig. 1, F4) ultimately leads to a curved flange section (Fig. 1, C), which produces the highest moment resistance, ductility and initial stiffness in this evolutionary process. The bent elements support each other by producing in-plane action, thus enhance the buckling resistance of the flanges.

FE results for development of a high performance CFS beam-column connection

A web bolted CFS beam-column connection (Fig. 2) was developed to avoid any out-of-plane forces and premature failure within the connection elements (Bagheri Sabbagh et al, 2011 and 2012). The main components are the crossed through plates passing between the separated beam and column sections and bolted to them.

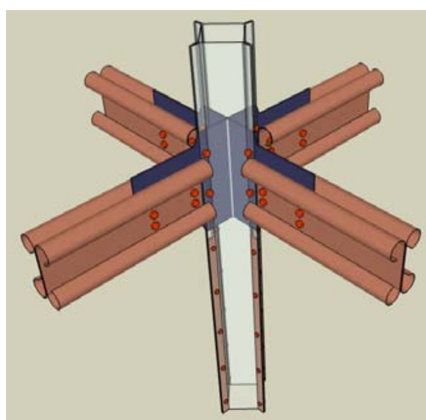

Fig. 2 Sketches of the web bolted through plate CFS beam-column connections

The inelastic behaviour of the proposed CFS beam-column connection with the through plate (TP) is investigated in this section by FE analysis. The FE model comprised a one-sided 2m length cantilever beam and 3m length column reflecting the inflection points of an external frame under lateral loading. Tied connections were used instead of the bolts between the beam webs, TP and the column lips.

The web bolted connection (without stiffener) produced 25% less moment strength (Fig. 3) than that of the same beam with fixed end connection (presented in the previous section). The reason is premature web buckling that occurred due to the lack of continuity in connection of the flanges which causes stress concentration in the webs at the first line of the beam-TP connection (Fig. 3). Different stiffener configurations were examined (three of them are shown in Fig. 3) to improve the moment-rotation behaviour. It was found that a combination of vertical and horizontal stiffeners (called as “optimum stiffener” in Fig. 3) produced the best results among the examined stiffeners such that the moment strength increased by ~40% compared with the connection without stiffener. Furthermore, the optimum stiffener connection produced no strength degradation up to the end of the analysis at 0.06rad rotation.

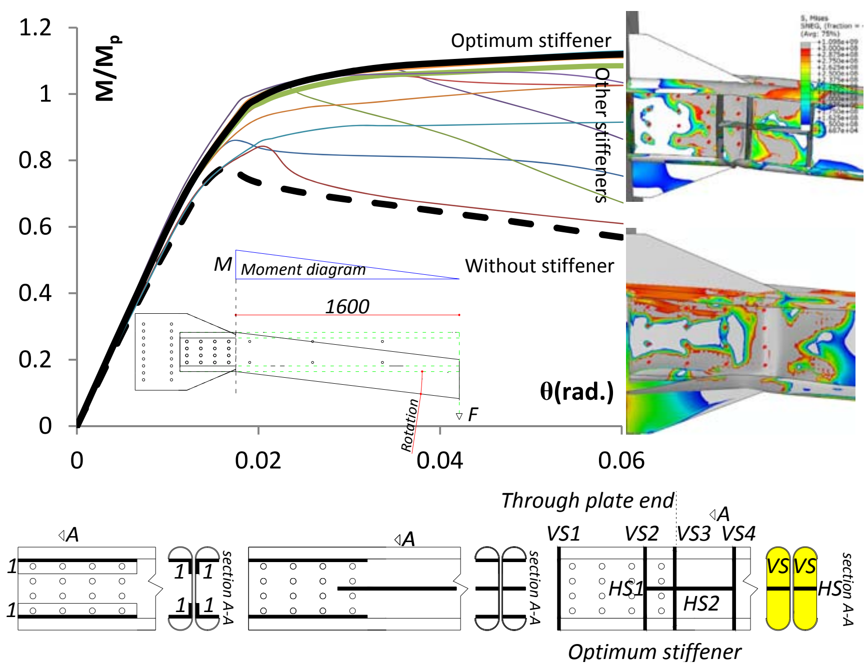

Fig. 3 Moment-rotation curves of the CFS web bolted beam-column connections with different stiffener configurations (M_p: nominal plastic moment of the beam with F_y= 275MPa)

In the optimum stiffener configuration two pairs of vertical stiffeners (VS1 and VS3) at the TP end and the beam end (called as "minimum stiffener") were used to delay the premature web buckling and avoid opening up the flanges, respectively as appeared in the connection without stiffener (Fig. 3). In addition, two more pairs of VS2 and VS4 were used at the distances of half the web height and the web height from the TP end, inside and outside the connection, respectively (in the range of local buckling wavelength). The horizontal stiffeners (HS1 and HS2) were also used to restrain VS2-4 and to avoid their deformation on the wave of the flange buckling.

The connections with optimum and minimum stiffeners were the two configurations used in the experimental work (presented in this paper) along with the connection without stiffener for bench marking.

Testing arrangements and set-up

Six beam-column connection assemblies (Fig. 4) were tested (Bagheri Sabbagh et al, 2012) with two different beam thicknesses of (A) 3mm and (B) 4mm and three connection stiffener configurations of without stiffener, with minimum stiffener and with optimum stiffener (A1-3 and B1-3). All the connections were designed following the slip-critical requirements of the bolted connections using AISC Specification for Structural Joints (2004). The reaction column comprised two hot-rolled channels to fit the test rigs instead of CFS sections used in the FEA as the CFS columns remained intact in the FEA.

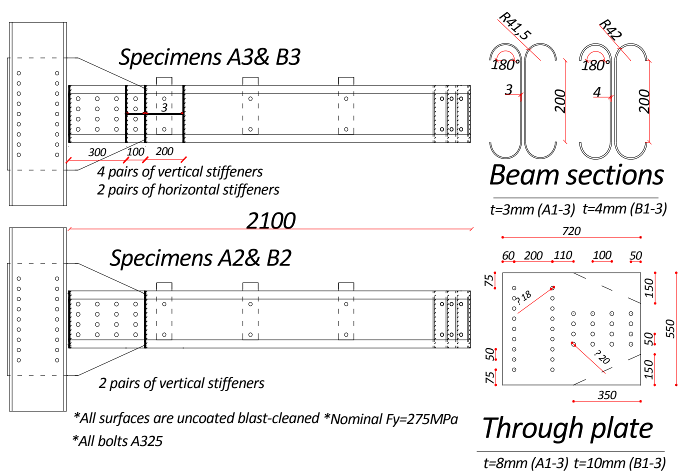

Fig. 4 Dimensions and configurations of beam-column connection assemblies

Cyclic loading was applied through a hinge connection at the beam end (Fig. 5) using a loading protocol given in AISC Seismic Provision (ANSI/AISC 341-05) for qualifying beam-column moment connections in special and intermediate moment frames. Lateral restraints were used at the plastic hinge and loading regions (Fig. 5) specified in the same provision (ANSI/AISC 341-05).

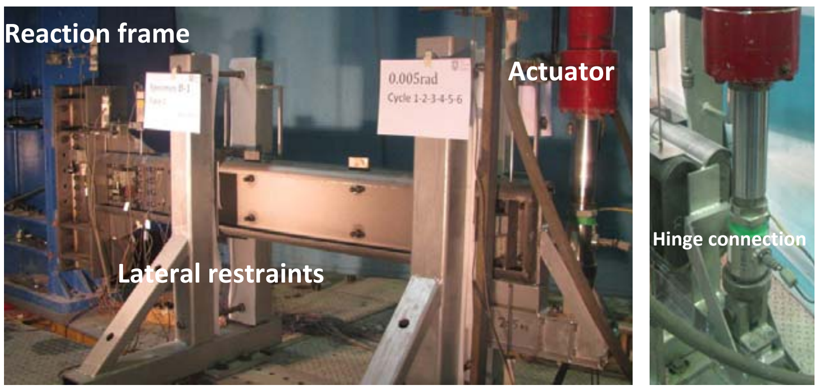

Fig. 5 Test set-up

Two types of moment-rotation behaviour were recognized for the tested connections: dominated by (i) rotation in the beam and (ii) rotational behaviour produced by slip-bearing action. The test results are described in the following sections.

Test results for the specimens dominated by rotation in the beam (Specimens A1-3 and B1)

The moment-rotation curves and failure deformations obtained for Specimens A1-3 and B1 dominated by flexural deformation and local buckling in the beam are shown in Figs. 6 and 7, respectively. Different regions can be identified in the moment-rotation curves: elastic region (*AB*), inelastic region (*BC*) leads to the maximum bending moments at Points C and C^-, postbuckling region (*CD*) leads to 80% of the maximum bending moments at Points D and D^-, failure region (*DE*) leads to the connection failure at Points E and E^-.

In Specimens A1 and B1, premature web buckling at the first line of the bolts of the beam-TP connection (initiated at 0.03rad and 0.05rad, respectively) as well as opening up the beam flanges at the connection region (Figs. 6 and 7) occurred similar to the FE results (Fig. 3). These highlight the need for at least two vertical stiffeners at the beam end and at the TP end (minimum stiffener).

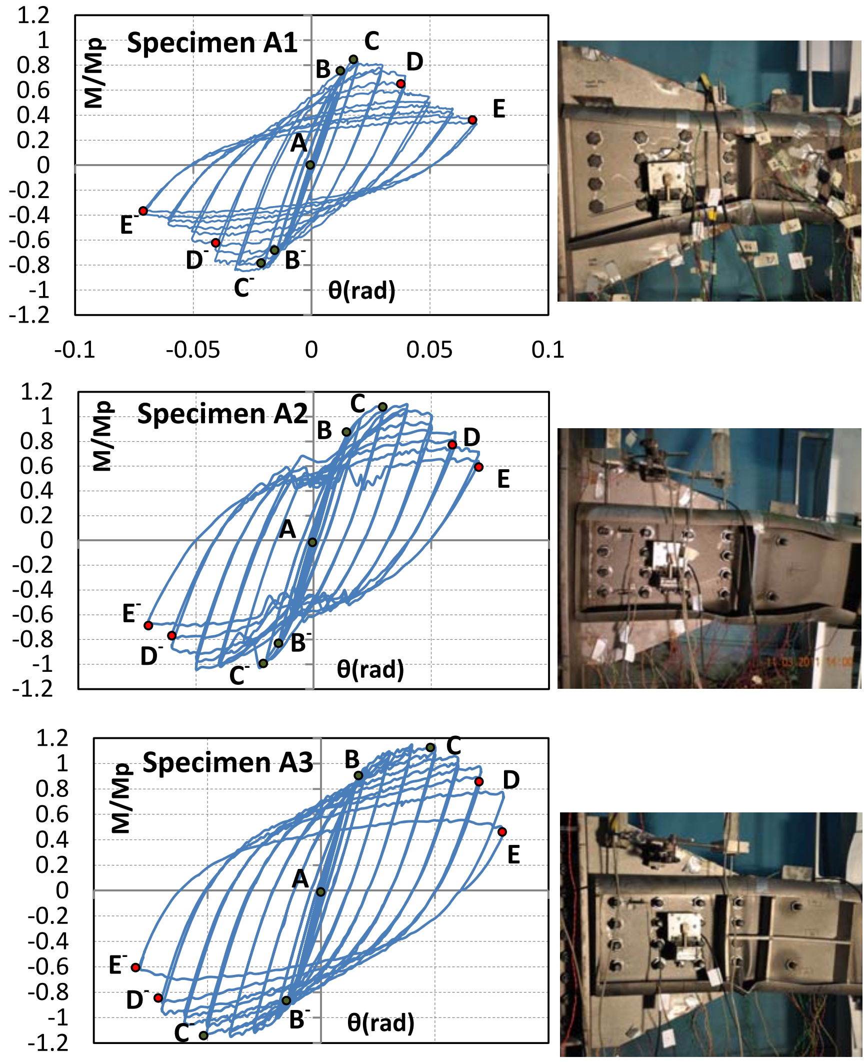

Fig. 6 Moment- rotation curves and failure deformations of Specimens A1-3

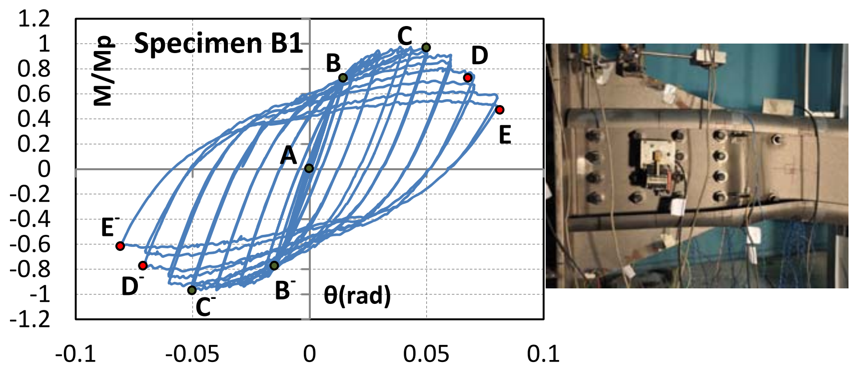

Fig. 7 Moment- rotation curve and failure deformation of Specimen B1

The ductility factor based on the rotation ratios at Points D and B increased by 50% and 75% for Specimens A2 (with minimum stiffener) and A3 (with optimum stiffener), respectively relative to A1. Correspondingly, the moment strength increased by 29% and 35% for Specimens A2 and A3, respectively compared with A1. The beams of Specimens A2-3 reached nominal plastic moment (M_p) and sustained large plastic deformation (similar to Class 1 cross sections in Eurocode 3, part 1.1), larger than 0.04rad at the level of $0.8M_p$ required for SMFs in AISC Seismic Provisions.

Test results for the specimens dominated by rotational behaviour produced by connection slip (Specimens B2-3)

The moment-rotation curves and the connection deformation at the last cycle of Specimens B2 and B3 are shown in Fig. 8. The regions correspond to different aspects of the connection behaviour are: elastic region (*AB*), prebuckling-slip region (*BC*) and postbuckling region (*CD*). In these specimens the design of the bolted connections was such that the required/available slip resistance ratio was slightly above 1.0 using the maximum moment obtained in the tests. Therefore, connection slip was expected. The slip-bearing action contributed around θ=0.05rad (measured by inclinometers in the connection region) in total rotation in the both specimens led to elongation of the bolt holes (Fig. 9).

The ductility factors increased by 28% and 43% and the moment strength increased by 10% and 23% for Specimens B2 and B3, respectively compared with Specimen B1. No strength degradation and failure occurred in both specimens except a slight distortion of the flange edges in Specimen B2 (Fig. 8).

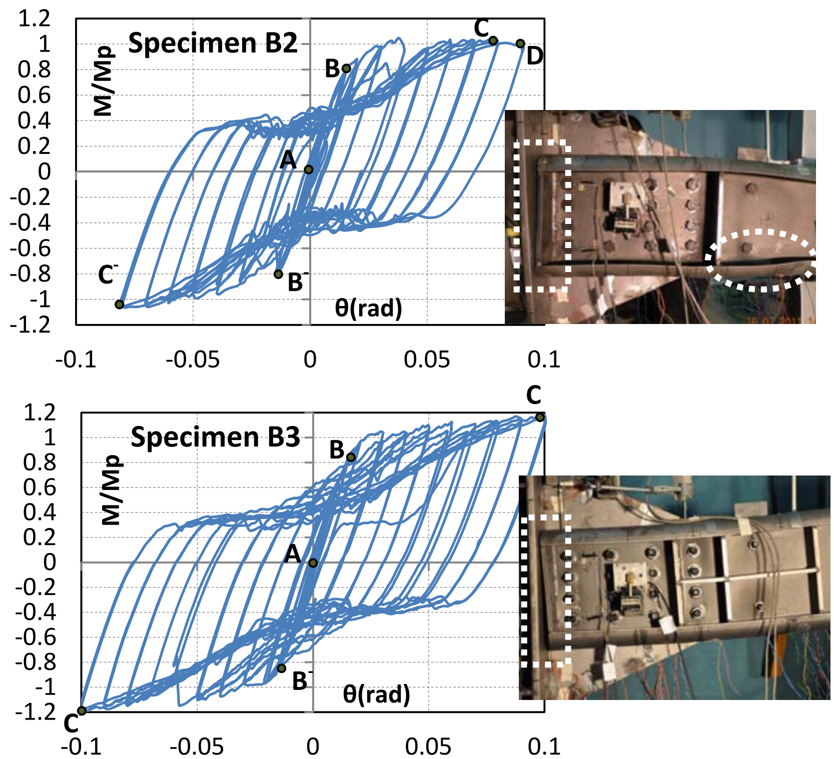

Fig. 8 Moment-rotation curves and connection deformation at the last cycle (Specimens B2 and B3)

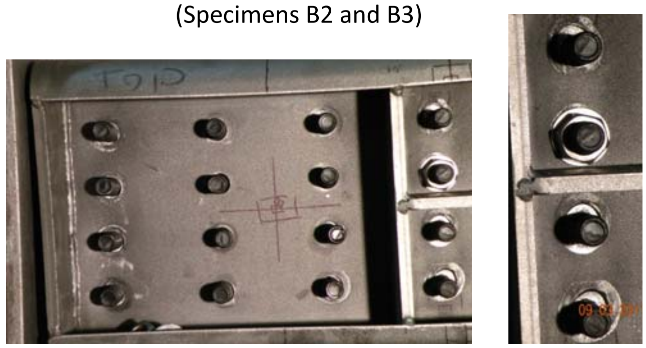

Fig. 9 Elongation of the material around the bolt holes (Specimen B3)

In order to retest the specimens after reaching very large rotation in the first test, the pretension forces of the bolts of the beam-TP (B-T) and TP-column (T-C) connections were increased (from 42% of the tensile strength of the bolts to 68% and 56% for B-T connections of Specimens B2 and B3, respectively and to 60% for T-C connections of both specimens).

The behaviour pattern for the retest of Specimens B2 and B3 (Fig. 10) was similar to that of the specimens dominated by rotation in the beam (presented in the previous section) without the elastic region. The connection slip was limited and the specimens were failed at Point E (Fig. 10) led to rupture of the flanges due to low cycle fatigue extended along the first line of the bolts (Fig. 11).

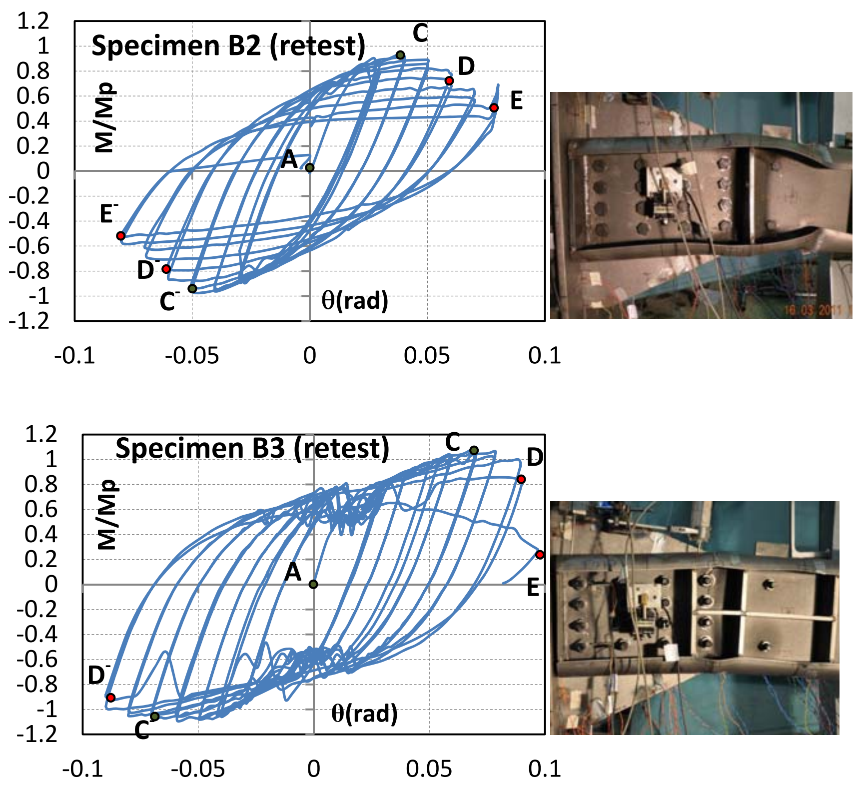

Fig. 10 Moment-rotation hysteretic curves and connection failure in retest of Specimens B2 and B3

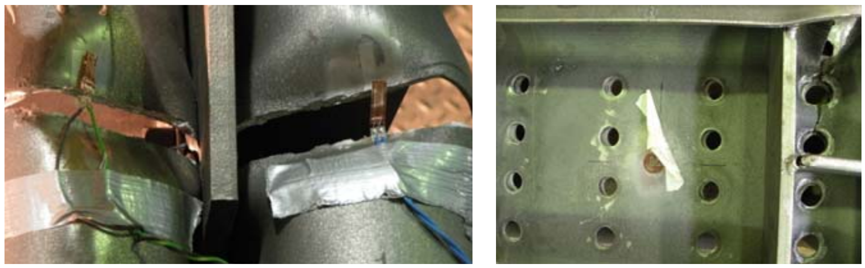

Fig. 11 Rupture at θ= 0.1rad cycle (Point E) in retest of Specimen B3

Hysteretic energy dissipation of the tested specimens

The cumulative hysteretic energy dissipation (*E*) of all the tested specimens is shown in Fig.12. The use of minimum stiffener for Specimen A2 and optimum stiffener for A3 increased the hysteretic energy by ~30% and ~90%, respectively and allowed larger rotation compared with A1. Highly stable hysteretic cycles were achieved for Specimens B2 and B3 by activation of connection slip which enables the specimens to reach very large rotations. This led to an increase in the hysteretic energy by ~240% for retest of Specimens B3 (with ~50% slip-bearing action contribution in total rotation in the first test) relative to B1 (with negligible slip). Connection slip can be utilised to minimise the damage and repair time of the main members in severe earthquakes required for Damage Control Structural Performance Level (FEMA 356, 2000).

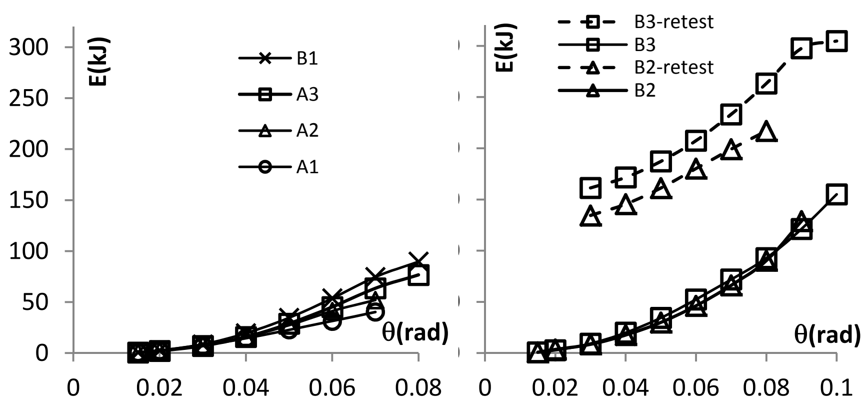

Fig. 12 Hysteretic energy dissipation curves of all specimens

Conclusions

The results of the FE investigation and experimental work showed that curved flange beams in conjunction with appropriate connection details can improve the moment-rotation behaviour in accordance with the requirements of Class 1 cross sections in Eurocode 3 and SMFs in AISC Seismic Provisions.

The through plate connection is suitable to produce in-plane action required for CFS sections with thin-walled elements. The premature web buckling due to discontinuity in the flange connection can be delayed by using a combination of vertical and/or horizontal out-of-plane stiffeners (called as “minimum and optimum stiffeners” in this study).

The use of minimum stiffener (for Specimens A2 and B2) and optimum stiffener (for Specimens A3 and B3) can increase the ductility factor by up to ~75%, the moment strength by up to ~35% and the hysteretic energy by up to ~240% also due to the activation of connection slip, compared with the specimens without stiffener.

By activation of connection slip highly stable hysteretic behaviour was achieved (in Specimens B2 and B3) which can also be utilised to enable high performance structural behaviour in severe seismic incidents.

Acknowledgement

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